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**DECEW GS HEADPOND
SLOPE STABILITY INVESTIGATION**

Report No 86-251-K

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ABSTRACT

During drawdown of the DeCew GS headpond for inspection purposes, a failure occurred in the headpond slopes near the St Catharines Water Works Canal. A soils investigation and slope analyses were performed to determine the strength properties of the native clay soils and to assist in the design of remedial works. By placing a berm of rockfill at the toe of the failed slope, the factor of safety will be raised to 1.22 under drawdown conditions, according to these analyses. The field investigation and analyses are described.

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To Mr. J.V. Doyle
Manager
Civil Works Department
Thermal and Hydraulic Generation Division

Attn: Mr. E. Percival

DECEW HEADPOND SLOPE STABILITY INVESTIGATION

INTRODUCTION

Civil Research involvement in the investigation of the DeCew Falls GS headpond slope failure came at the request of Mr. E. Percival of the Civil Works Department. A soils investigation, composed of field and laboratory testing, and slope stability analyses were performed as a result of this request. The stability analyses produced factors of safety for the slope in its current condition and with remedial measures taken. Both normal and drawdown water levels were used in these analyses. The remedial work was expected to consist of building a rock toe berm to maintain an acceptable factor of safety during rapid drawdown conditions.

BACKGROUND

Failure of the slope investigated in this study occurred while the headpond was being dewatered for inspection purposes. The location of the circular slope failure on the DeCew Falls GS headpond (known as Lake Gibson) is shown in Figure 1. The location of the slip surface was defined by the exposed scarp on the slope as shown in Figure 2. The exit of the slip surface was not observed because it was below the water level of Lake Gibson. The failure did not affect the adjacent St Catharines Water Works Canal or the road that separates these two bodies of water. There was a Bell Canada cable that traversed the failure area which experienced some movement and damage to its concrete encasement but the cable remained functional. There was no witness to the failure. The location of failure was discovered the following day during a routine inspection of the slopes conducted each time the water level is drawn down.

SOILS INVESTIGATION

Investigations included sampling, identification and field testing of soils both on shore near the failure scarp as shown in Figure 3, and in the area off shore from the slope failure. Field testing consisted of one standard borehole and one dilatometer hole on shore and six dilatometer holes off shore. The dilatometer is an in situ testing tool developed by

Marchetti/1/. Testing in the laboratory included identification, moisture content, undrained shear strength, density, Atterberg limits, grain size analysis, triaxial (drained shear strength) tests and permeability tests.

Samples were taken on shore at the location marked BH2 in Figure 3, using hydraulically pushed Shelby tubes. Field vane testing at selected depths followed the Shelby tube sampling. The geotechnical data sheet, Figure 4, contains a record of these drilling and testing activities. The shear strength profile for the soil is shown in Figure 5. Standard laboratory tests were performed on selected soil samples recovered from the Shelby tubes. Grain size distributions are shown in Figure 6. Atterberg test results are shown in Figure 4. Triaxial test measurements are graphically displayed in Figure 7. Triaxial permeability results are tabulated in Table I.

A dilatometer was pushed into the soil adjacent to BH2 on the shore behind the failure scarp. Results for this test, labeled BH1, are included in Appendix A, with the interpreted shear strength values also included on Figure 5. The dilatometer was selected because it can gather a relatively large amount of data over a closely spaced interval. It was believed that this would be a relatively inexpensive way to assess soil parameters in the area offshore.

Dilatometer readings were taken in the offshore area at the locations indicated in Figure 3. The dilatometer was pushed into the sediments by a technician leaning over the side of a small boat. In addition, sediments were grab sampled from the lake bottom for chemical analyses. The results of these analyses were not available at the time of printing and will have to be reported later.

An inclinometer casing was grouted in BH2 to permit future monitoring of this slope. Civil Works will take initial readings on this device.

SUBSOIL CONDITIONS

Before starting this investigation, it was known that clay dykes were built to contain Lake Gibson when DeCew GS was originally constructed, circa 1898. New clay dykes were created over the old dykes when the level of the headpond was raised in the late 1940's. Natural clays were expected to exist below the dykes.

In the onshore test locations a layer of stiff to hard silty clay was found overlying a deposit of firm clay. The upper silty clay, which extends to a depth of about 14 feet below road level, corresponds to the new clay dyke, described above. This layer has shear strengths from 2000 to over 5000 psf. The firm clay, which extends to bedrock at 35.5 feet, generally has shear strengths between 1000 and 2000 psf. Fractures are extensive in the upper one foot of the limestone bedrock immediately below the clay as indicated by the loss of water during drilling. The quality of the bedrock was much better below this level.

In the offshore test holes the dilatometer could only be advanced (by hand) to depths ranging from 1 to 4 feet. It is believed that the depth at which "refusal" was met represents the top of the competent natural soil and that the material probed by the dilatometer was "mud" or lake bottom sediment. This sediment is very weak, having shear strengths in the range of 30 to 40 psf.

The undrained strengths obtained from the field vane and the dilatometer are normally in quite close agreement/2/. The undrained strengths, for this site shown in Figure 5, show considerable scatter. This may be caused by the fissured clay at the site, which tends to behave like a granular material when strained to failure. The method of assessing soil properties with the dilatometer in the offshore area using the boat did not give conclusive results because the motion of the boat would cause localized failure of the soil along the side of the dilatometer. In addition, it was not possible to apply sufficient force to the dilatometer to obtain readings within the native soil below the bottom sediments. The ranges of values for selected measured soil properties and some from the literature are listed in Table II along with the values used in the stability analysis.

SLOPE STABILITY ANALYSIS

When back-analyzing a slope failure, measured values of soil properties serve as a starting point in the slope stability calculations. But, these values may have to be altered to obtain a safety factor of 1.0 or less so that the behaviour of the mathematical model matches the observed behaviour of the real slope. Once the model has been calibrated in this fashion, it may be used to predict the performance of the real slope under other conditions. In this case, these other conditions included slope stability with headpond water level at normal elevation and with a remedial rockfill toe berm in place.

The slope failure occurred during the period when the water level in Lake Gibson was drawn down approximately 17 feet. The slope was analyzed using both total stress and effective stress analysis for normal and rapid drawdown conditions. Effective stress analysis was most appropriate for this slope because it allowed the destabilizing water pressures in the slope, caused by drawdown, to be properly taken into account. Two flow nets were used but the flow pattern did not significantly affect the calculated factor of safety. Efforts were made during analyses to adjust the soil property values until a failure would just occur.

Using an undrained strength of 3000 psf for the upper clay (fill) layer and 1400 psf for the lower clay yielded a safety factor of 4.1 for a total stress analysis of the slope with the headpond dewatered. Reducing these strengths to 2000 psf and 500 psf for the upper and lower layers respectively and repeating the above analysis reduced the safety factor to 1.8. As the clay had a sensitivity of about 2 to 3, these reduced strengths correspond to the lower bound undrained or residual shear strength of the clay. These calculations showed that total stress analysis would not produce a factor of safety near 1 (representing failure) while

using realistic strength parameters. A change to effective stress analysis was therefore warranted.

The shear strength characteristics of the soils used for the effective stress analyses are shown on the cross section presented as Figure 8. The cross section survey was provided by Civil Works. The upper 14 feet of soil measured from the top of the slope is fill material that was placed and compacted over the weaker native soil. The upper level of clay soil is badly fissured and it has been assigned a cohesion value of zero on that basis. The value of effective friction angle for the lower, native clay soil was selected to be 24.5°, based on the results of an extensive in situ study undertaken by Kwan/3/ prior to the realignment of the southern half of the Welland Canal. The cohesion value of zero was found to be appropriate for the analysis of the DeCew failure because it was the only value that would produce a slope failure (ie safety factor = 1) during a rapid drawdown condition as shown in Figure 8. Note the failure surface location matches field observations. A laboratory triaxial test on DeCew material (Figure 7) confirmed that Kwan's shear strength parameters were reasonably representative of this soil and hence appropriate for this analysis. The slope stability analysis for normal lake level conditions, shown in Figure 9, yielded a 1.42 factor of safety value.

REMEDIAL WORK

It has been suggested that the failure slope be stabilized by the construction of a toe berm on the slope. The restriction on the toe berm is that it should be below the normal lake level and with a side slope of 2 horizontal to 1 vertical. Two designs for the toe berm are presented and analyzed for stability under normal and rapid drawdown conditions with the results summarized as follows:

<u>Elevation of the Top of the Berm</u>	<u>Normal Lake Level</u>	<u>Drawdown Conditions</u>
555'	1.415 (Figure 10)	1.239 (Figure 11)
550'	1.387 (Figure 12)	1.221 (Figure 13)

The proposed method of construction for either of the toe berm designs would involve 8 to 12 inch diameter angular rock, known as primary blast, being end dumped to form a road out into the water at the location shown on Figure 14. The width of the road would be approximately 10 feet to support a dump truck. The side slopes for this road would initially be steeper than 2 horizontal to 1 vertical. A backhoe would then proceed to dig the primary blast down to the design elevation dumping the excavated rock into the water alongside of where the road had been. This action would widen the toe berm and reduce the side slopes. During the placement of the rock for the toe berm, there will be some displacement of the softer sediments. The sediments may be squeezed into the rock toe berm and perhaps provide some lubricating action between the rock units. A geotextile fabric could be placed on top of the soft bottom sediments to function as a separator prior to the end dumping of the rock. This would be a better approach than

attempting to excavate the soft materials due to cost and environmental considerations.

The stability of the shoreline bank will not be affected by any movement or localized slope failure on oversteepened sections of the toe berm because the weight of the rock would still offer a resisting moment to the weight of the material within the slip circle on the bank. Note that the critical circle for the repaired shoreline passes only through the berm and does not intersect the original slope.

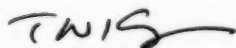
The choice as to which design elevation for the toe berm, since both ensure stability of the bank, becomes an economic decision. The cost for the extra rock for the 555 foot elevation must be compared with the extra handling cost of excavating the additional 5 feet with the 550 foot elevation design. Other concerns may involve navigation of pleasure craft which use Lake Gibson.

SUMMARY

The slope failure along the bank of the DeCew GS headpond occurred during the rapid drawdown of the headpond. Damage was confined to the slope and to the concrete cable duct for a Bell Canada cable. The adjacent roadway and St Catharines Water Works Canal were not affected.

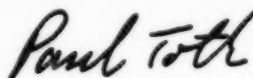
Soil investigations were carried out on shore and offshore using laboratory and field techniques. Based on the results of these investigations and those by others in this immediate area, slope analyses showed this slope to be in a state of limiting equilibrium under drawdown conditions. The existing slope was found to be stable with the headpond at normal operating level. Further analyses showed that installation of a rockfill toe berm would ensure the stability of this bank under future drawdown conditions. It is suggested that a geotextile be employed in the berm construction. An inclinometer was placed for future monitoring of this slope.

Approved by:



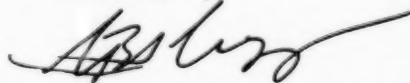
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Senior Engineer
Soil Sciences Section

T.A.C.

PT:LM

REFERENCES

1. Marchetti, S. "In Situ Tests by Flat Dilatometer". ASCE Journal of the Geotechnical Engineering Division, Vol 106, No GT3, March 1980.
2. Bechai, M. "Assessment of In Situ Testing Methods for Geotechnical Investigations of Transmission Lines". Ontario Hydro Research Division Report No 86-143-K, June 1986.
3. D. Kwan. "Observations of the Failure of a Vertical Cut in Clay at Welland, Ontario". Canadian Geotechnical Journal, Vol 18, p 283, 1971.

TABLE I

Triaxial Permeabilities for Selected Depths at Borehole 2

<u>Depth (feet)</u>	<u>Permeability (cm/sec)</u>
17-19	5.0×10^{-7}
23-25	5.3×10^{-8} to 1.0×10^{-7}

TABLE II

Ranges of Selected Soil Properties

	<u>Measured Lab or Field</u>	<u>Literature*</u>	<u>Value Used for Analyses</u>
Natural Density			
0 to 14 ft	123 - 129 pcf	115 - 127 pcf	127 pcf
14 to 35 ft	112 - 128 pcf	110 - 125 pcf	125 pcf
Vane Field Strength			
0 to 14 ft	1160 - 5520 ⁺ psf	1900 - 3300 ⁺ psf	
14 to 35 ft	1550 - 3750 psf	900 - 1800 psf	
Lab Vane Strength			
34 ft	960 psf		
Pocket Penetrometer			
0 to 14 ft	2260 - 4510 psf		
14 to 35 ft	770 psf		
C'			
0 to 14 ft		200 psf	0
14 to 35 ft		280 psf	0
φ'			
0 to 14 ft		21.5°	32°
14 to 35 ft	25° - 26°	24.5°	24.5°

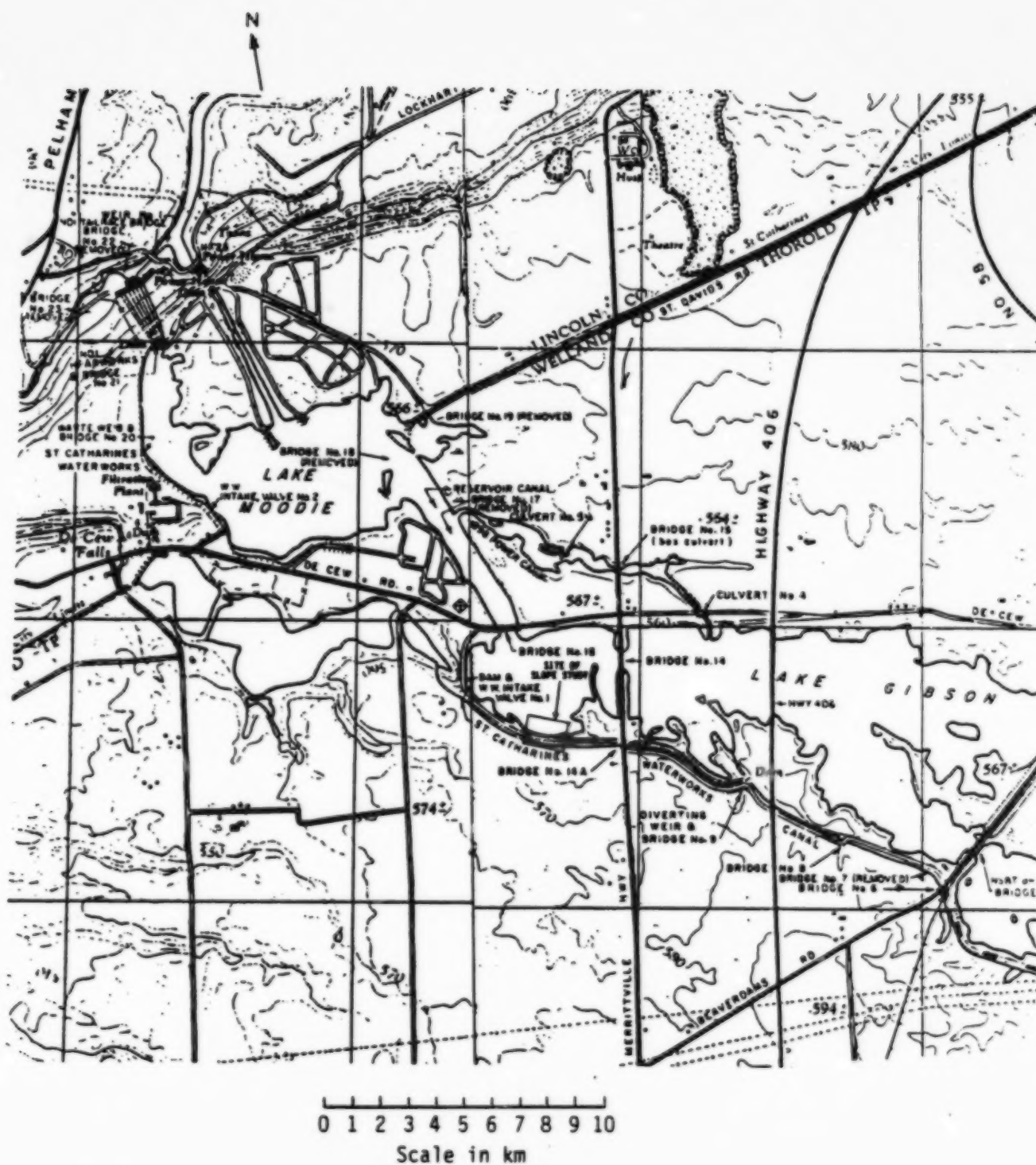


FIGURE 1
LOCATION OF SLOPE STABILITY INVESTIGATION

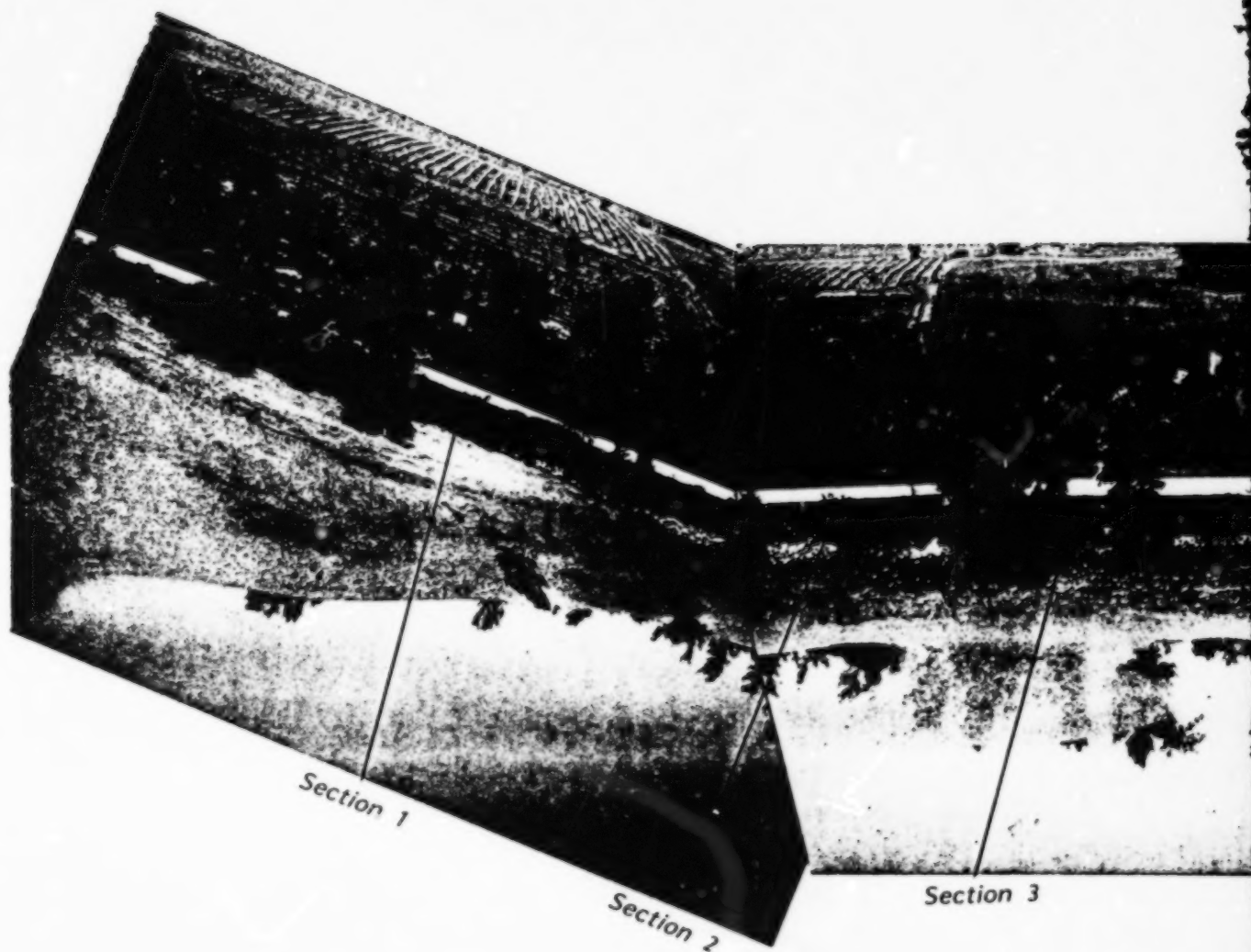


FIGURE 1
PHOTOGRAPHED FAILURE SURFACE



Section 4



Section 5

RE 2
RFACE DURING DRAWDOWN

Rib = Round Iron Bar
 ● Dilatometer Location
 ○ Field Vane and Shelby Tube

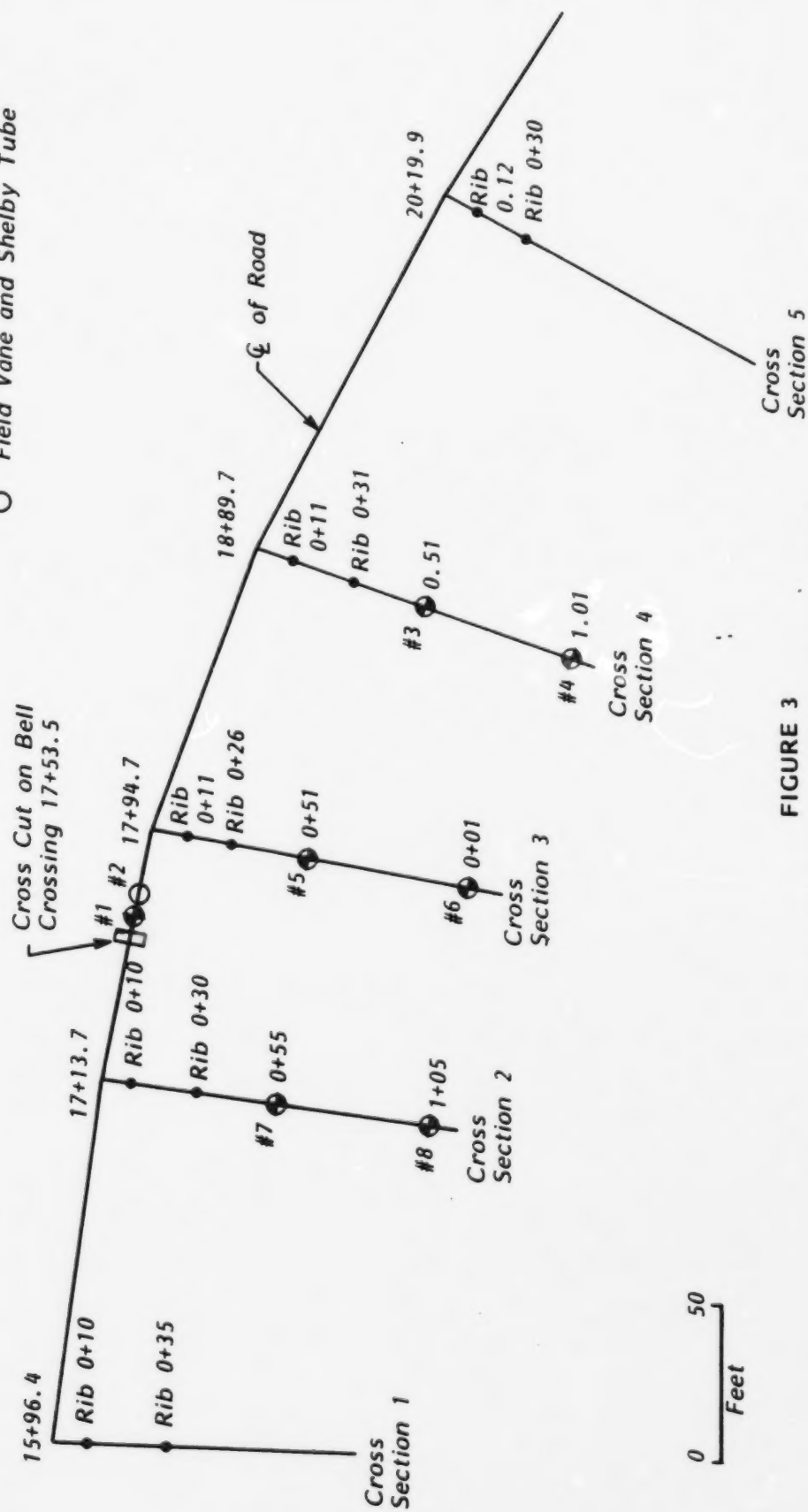


FIGURE 3
 PLAN OF LAKE GIBSON SLOPE STUDY

PROJECT . DECEW FALLS GS. SLOPE FAILURE	HOLE TYPE . HOLLOW STEM	OBSERVER . AM
LOCATION . SEE FIGURE 3	HOLE SIZE . 8-INCH	COMPILED . PT
. DIP	DRILLED . JULY 3, 1986	CHECKED . CBC
DATUM BEARING	DRAWN	APPROVED

BORING DATA				BORING LOG		WATER INF.		FIELD TESTS		LABORATORY TESTS									
SCALE - METRES ELEV DEPTH	(1) SAMPLE NO	(2) SAMPLE TYPE	(3)	(4)	(5) DESCRIPTION	(6) REMARKS	(7) OBSERVATION WELL DATA	(8)	(9)		NATURAL DENSITY .pcf	MOISTURE CONTENT - %	(10)		(11)				
			'N' VALUE per 0.3m	WATER LEVEL and Casing W.R.				soil/rock cm/s	SPECIAL TESTS	RESULTS TV(psf)			TYPE	STRENGTH psf	RESULTS	TYPE	RESULTS		
2	B1	pushed			Fill, crushed stone to 1 ft then silty clay, stiff to hard, brown, trace of fine sand					FV	5520+	2065	18.7	PP	4510				
4													2056	19.6	PP	4140			
6	B2	pushed																	
8											FV	1660		23.7	PP	2260			
10	B3	pushed																	
12	B4	pushed				12'					FV	5520+	1968	29.2	QU	14.50			
14						Clay, some silt to silty, firm, grey, a few silt partings					FV	1930	1980	29.0	QU	1120			
16	B5	pushed																	
18	B6	pushed										FV	3590	2013	25.5	QU	1670	LL	30.2
20																		PL	17.6
22	B7	pushed										FV	3750					PI	12.0
24	B8	pushed												2095	23.3	LJC	1380		
26											FV	1550							
28													789	47.7	QU	730	LL	63.6	
30											FV	1660-	844	37.4	QU	1210	PL	21.8	
32																	PI	31.8	
34																			
36	B9	pushed									FV	1550	1990	26.0	QU	1730			
38												27.2							
40	B10	pushed			35.5'							2055	24.6	PP	770	LV	960		
42	NXL	core			Bedrock, Limestone, top fast fractured, sound below														
					40.5'														
					END OF BOREHOLE														

on map

1 of 1

GEOTECHNICAL DATA SHEET

hole no
2

Fig 4

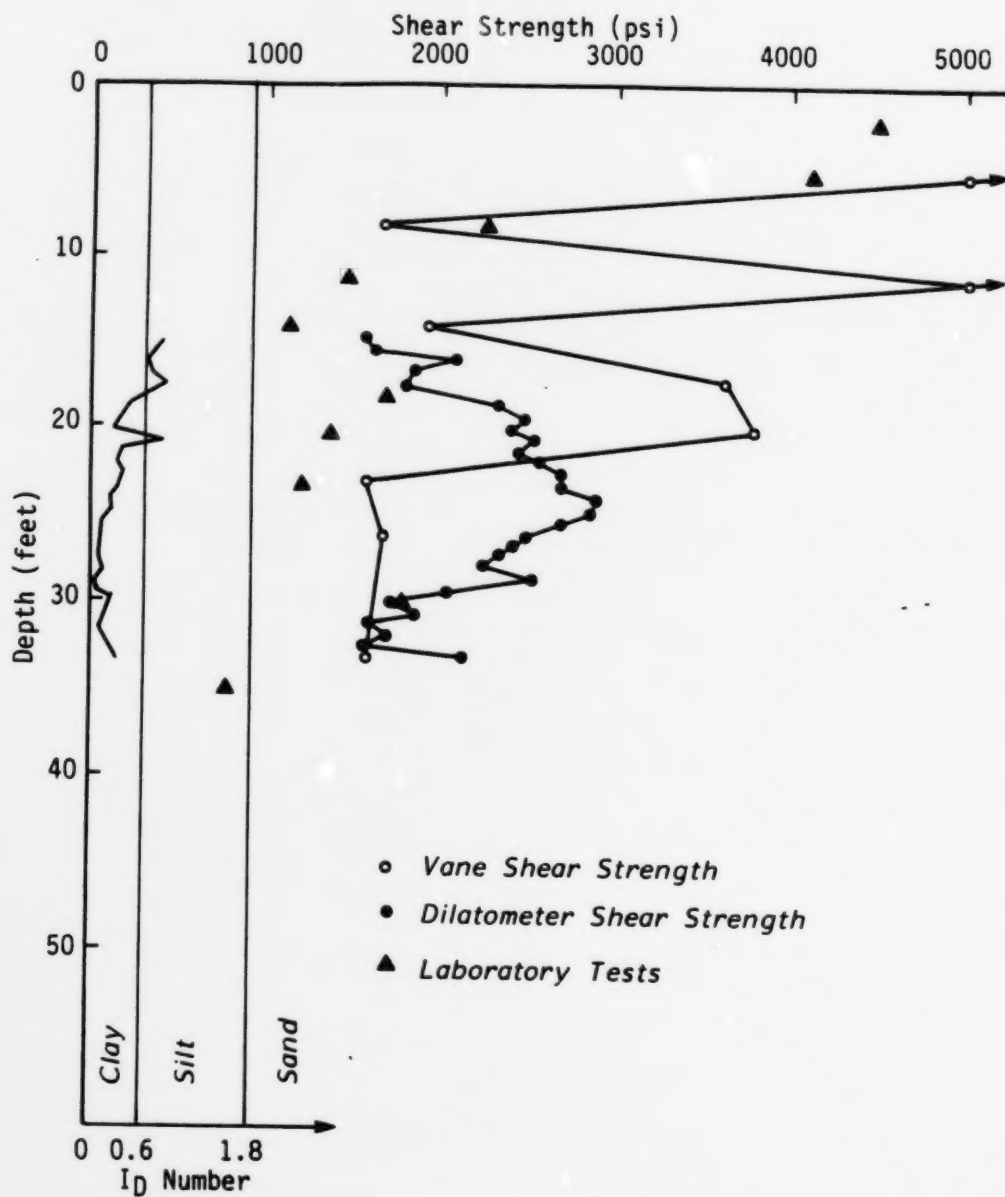
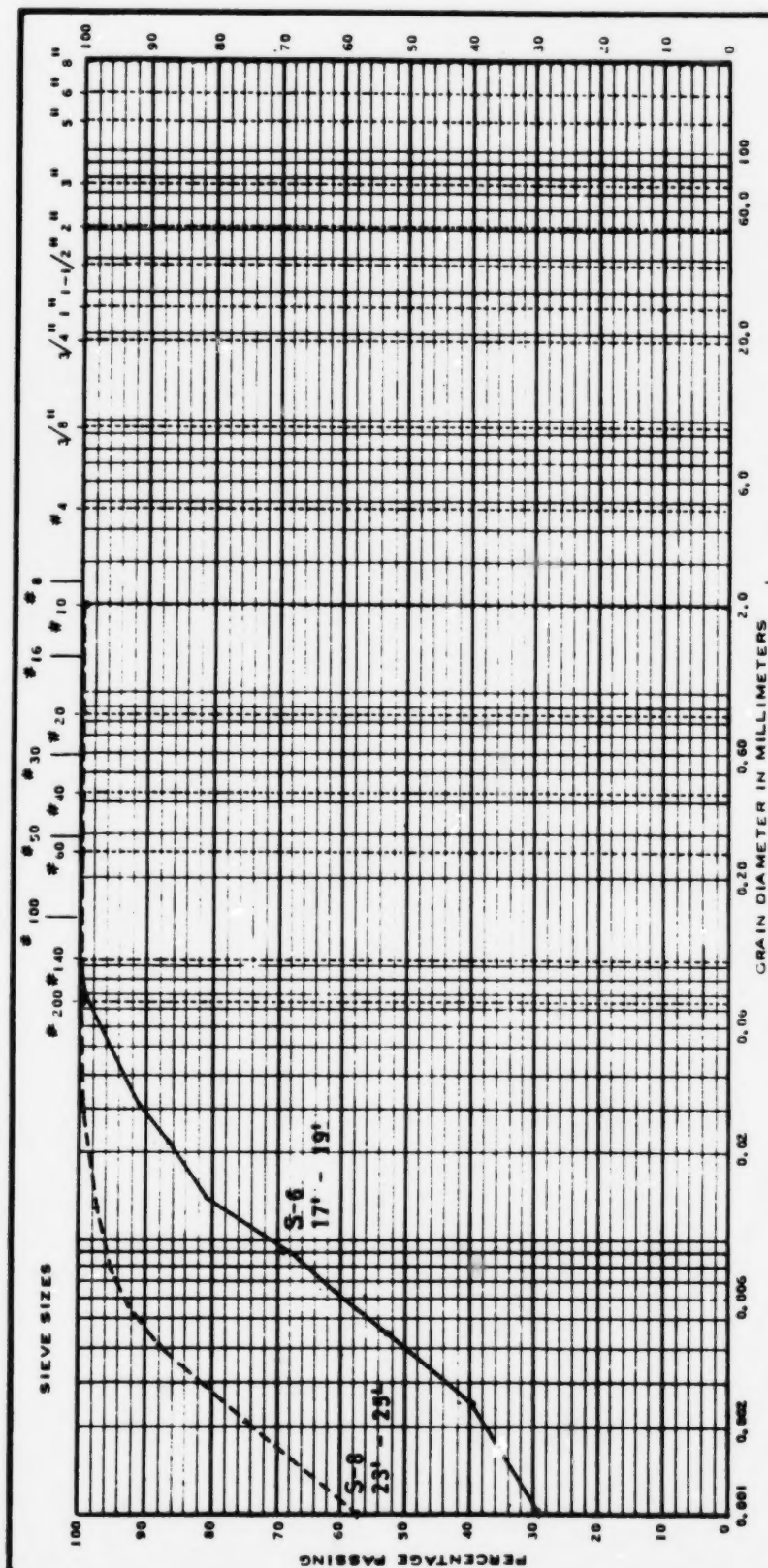


FIGURE 5
LAKE GIBSON SLOPE STUDY
SHEAR STRENGTH PROFILE AT BOREHOLES 1 & 2

MECHANICAL ANALYSIS



CLAY	TO SILT		SAND		GRAVEL		COBBLES
			FINE	MEDIUM	COARSE	FINE	COARSE

UNIFIED SOIL CLASSIFICATION

FIGURE 6

DECEW HEAD POND - BH2

90092
new 80-11

Ontario Hydro
RESEARCH DIVISION

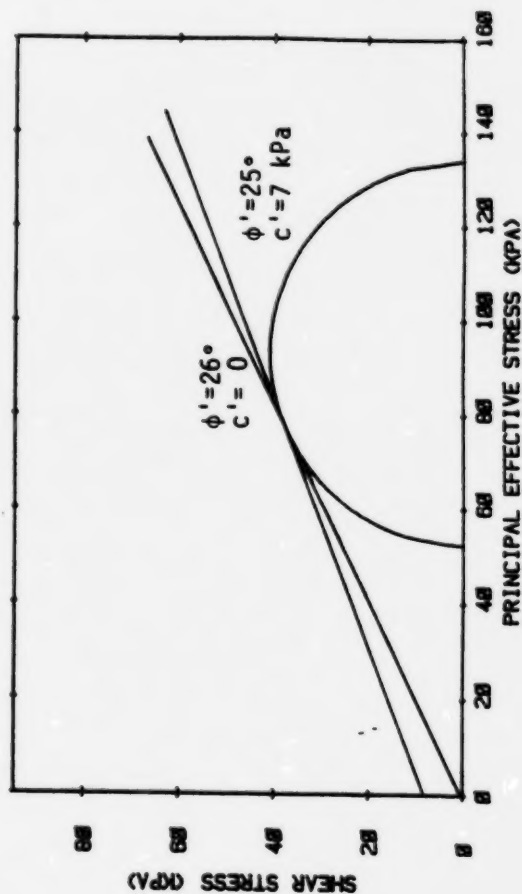
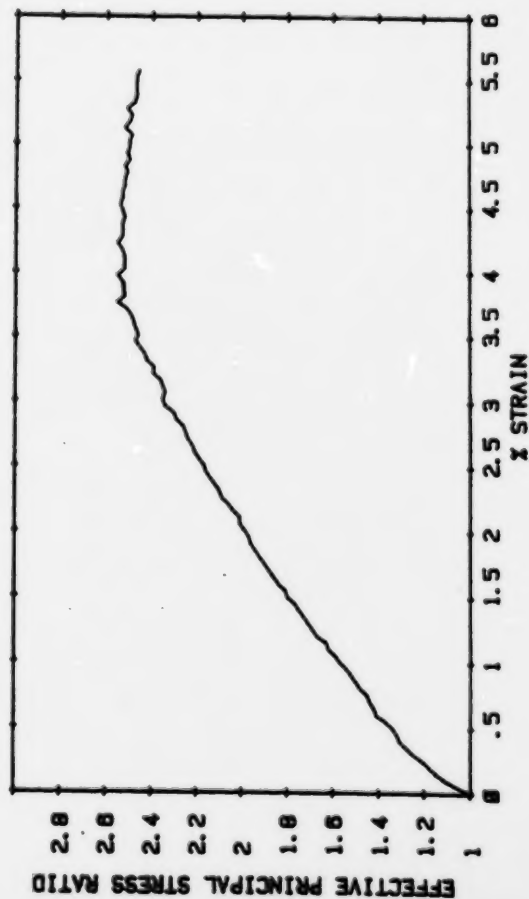
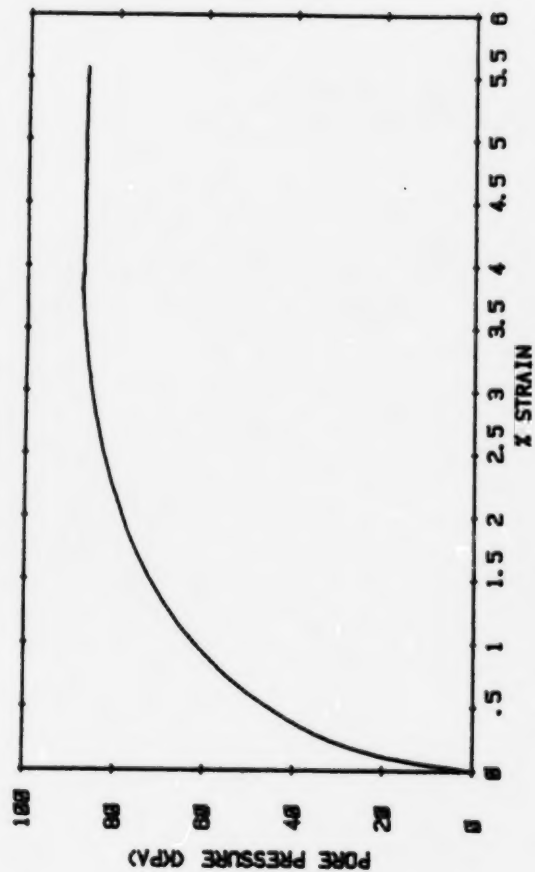
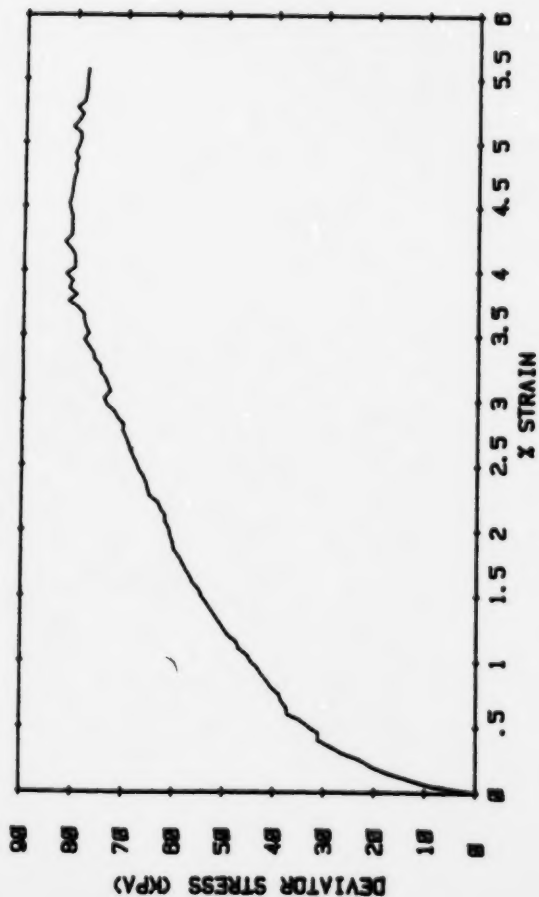


FIGURE 7
TRIAXIAL TEST IN NATIVE SOIL 20-22 FEET

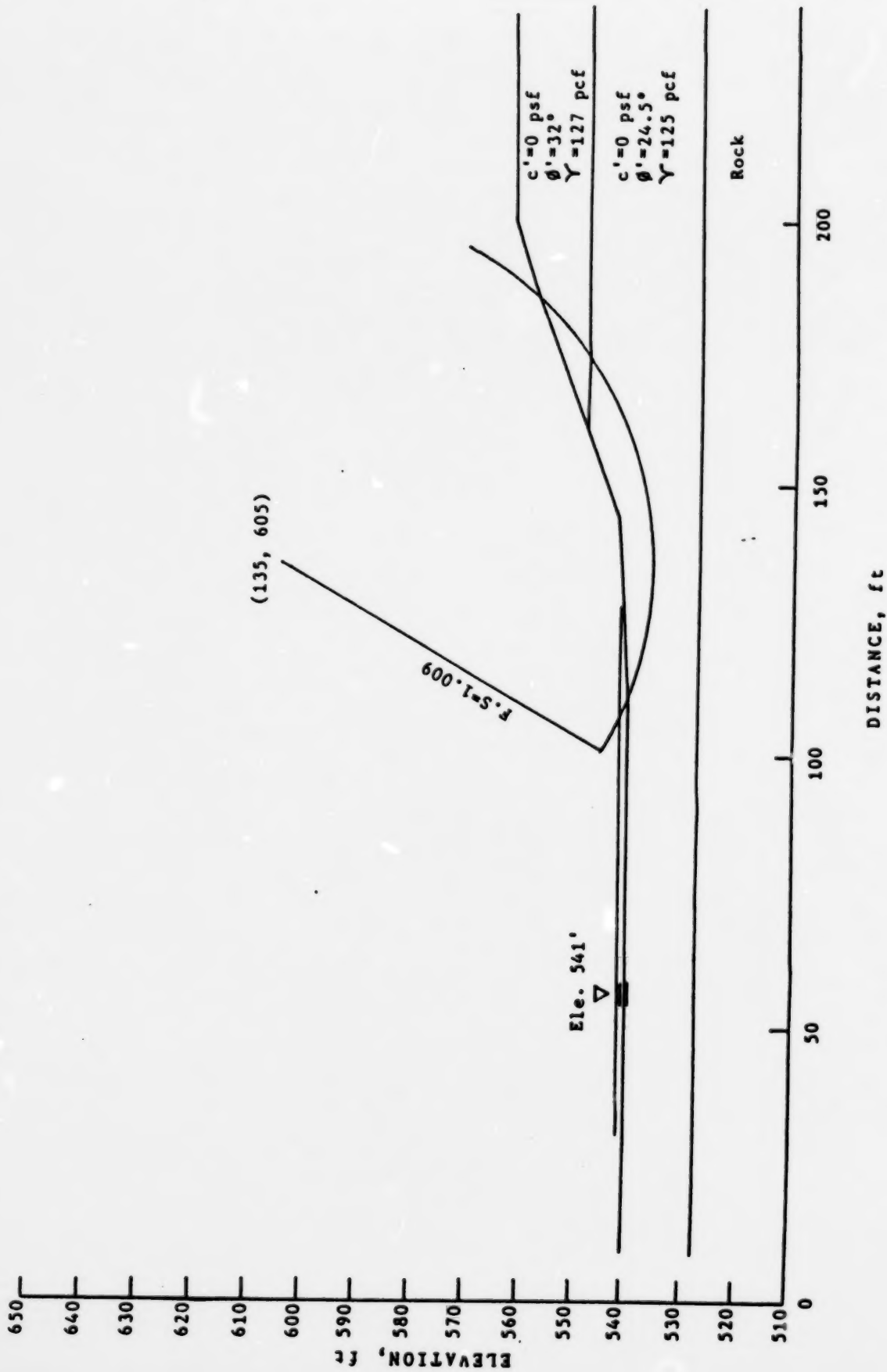


FIGURE 8
NATURAL SLOPE - DRAWDOWN CONDITION

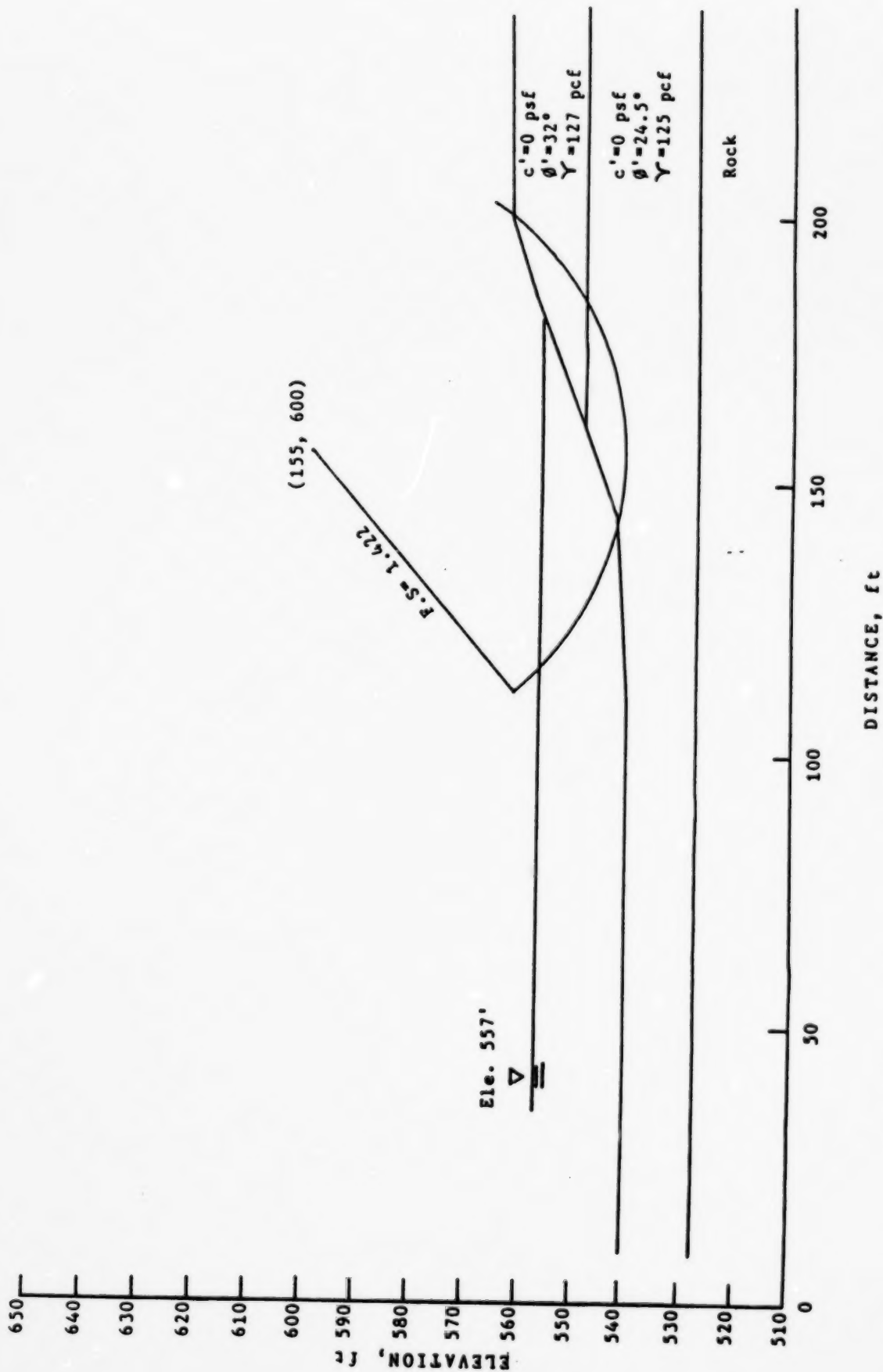


FIGURE 9
 NATURAL SLOPE - NORMAL LAKE LEVEL

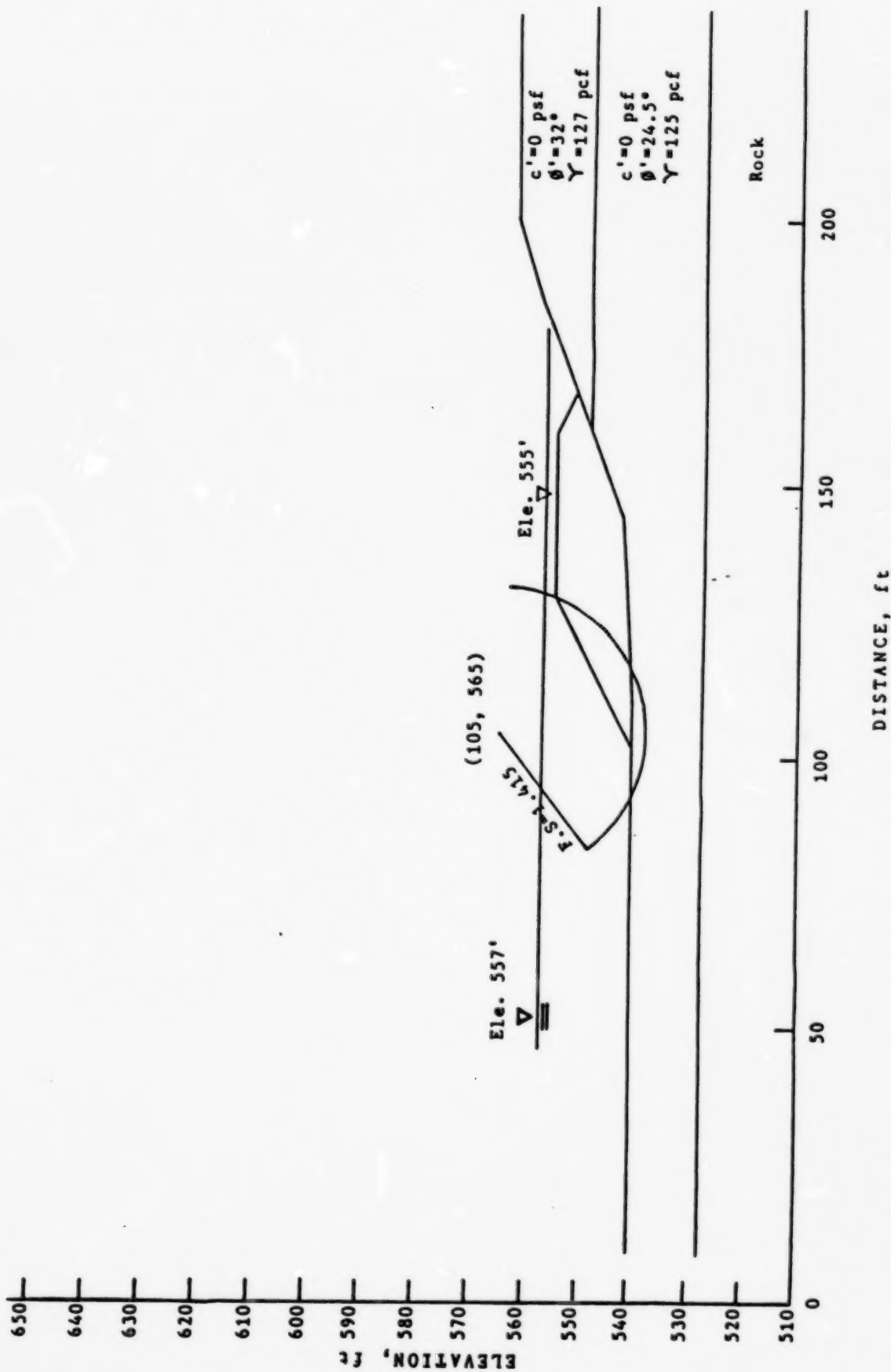


FIGURE 10
TOE BERM AT ELEVATION 555' - NORMAL LAKE LEVEL

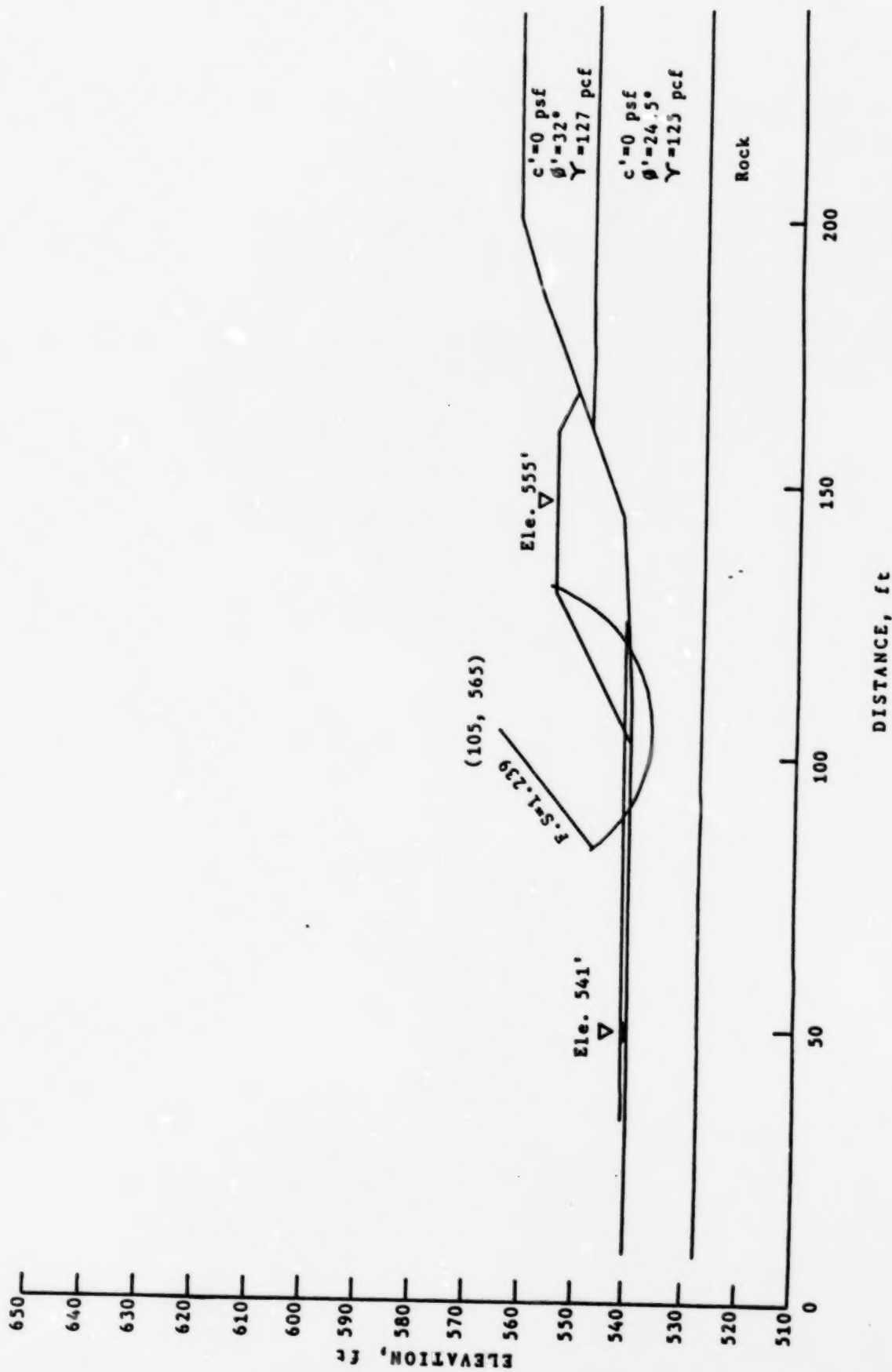


FIGURE 11
TOE BERM AT ELEVATION 555' - DRAWDOWN CONDITION

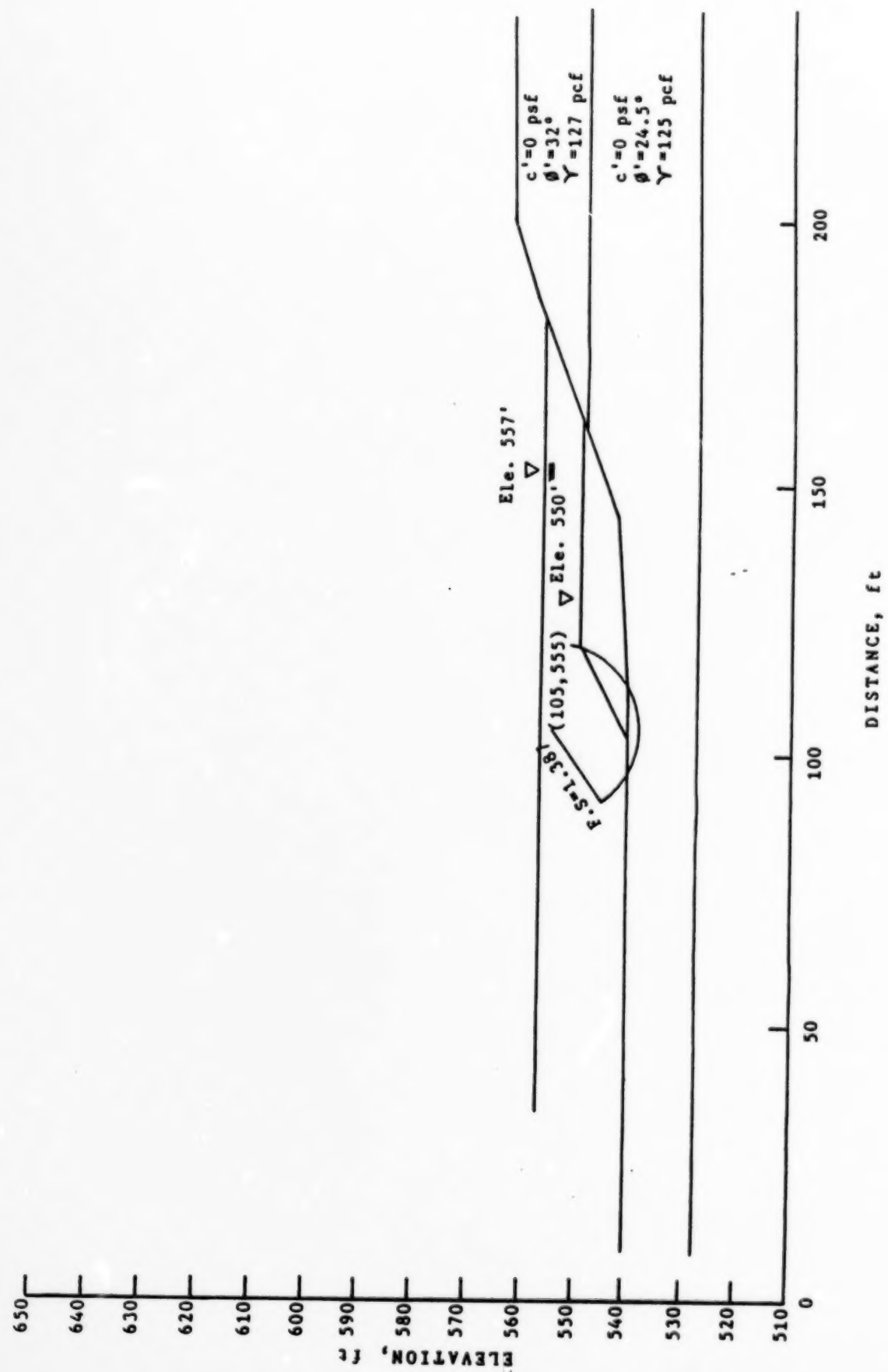


FIGURE 12
TOE BERM AT ELEVATION 550' - NORMAL LAKE LEVEL

172366-RD

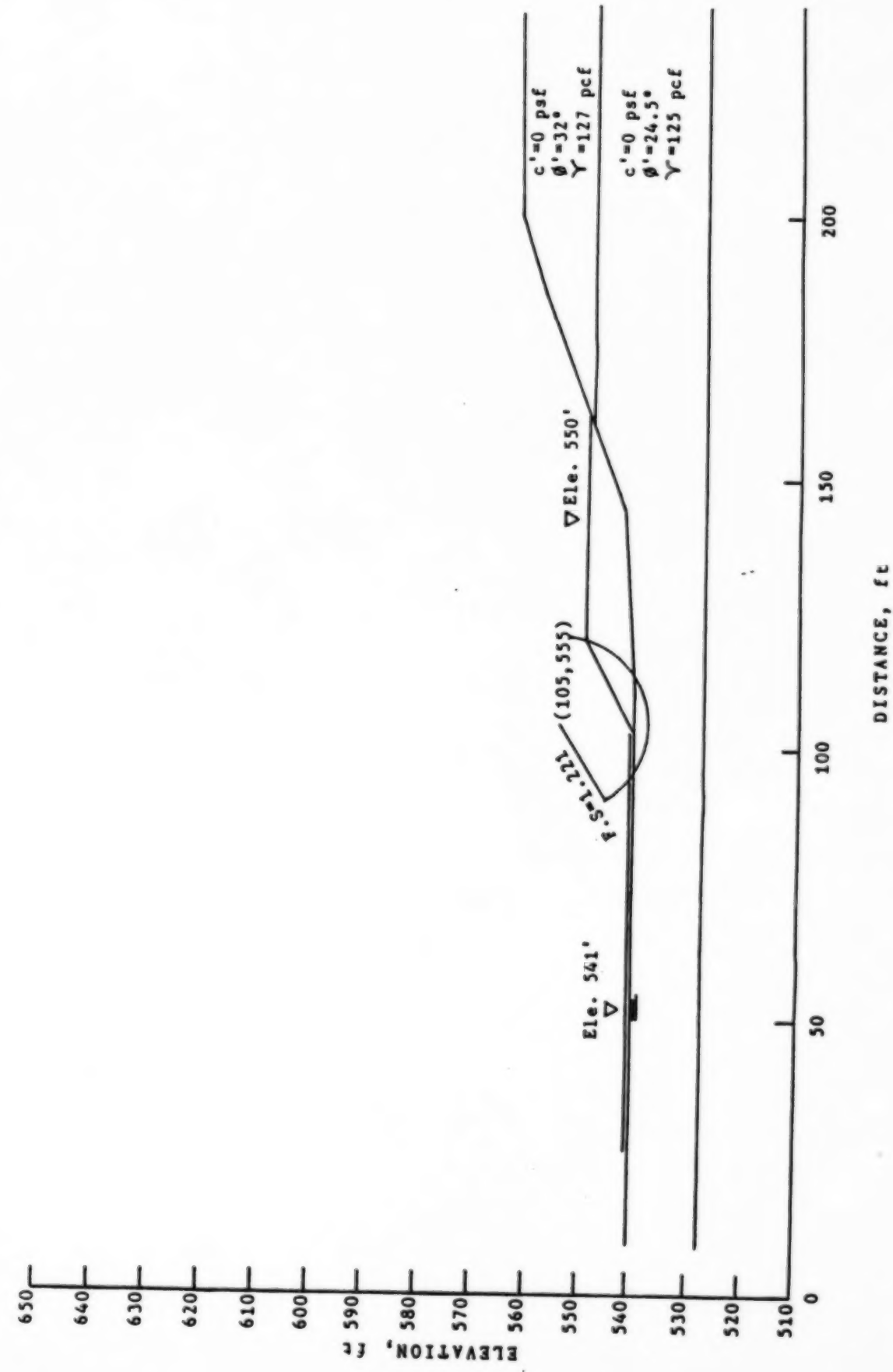


FIGURE 13
TOE BERM AT ELEVATION 550' - DRAWDOWN CONDITION

172367-RC

X-UNDRAINED SHEAR STRENGTH (CU) - BARS		X-PRECONSOLIDATION PRESSURE (PC) - BARS		X-MODULUS FOR 1-0 CONSOLIDATION M - BARS	
DEPTH	0.....1.....2+	0.....5.....10+	2- . . 5 . . . 10	20 . . 50.....100	200 . . 500+
5.00 M	+	+	+	+	16.4 FT
5.23 M	X	+	+	+	17.2 FT
5.44 M	X	+	+	+	17.8 FT
5.60 M	+	+	+	+	18.4 FT
5.80 M	+	+	+	+	19.0 FT
O-FRICTION ANGLE (PHI) - DEG		X-VERTICAL EFFECTIVE STRESS (SV) - BARS			
25-..30...35...40...45+	0.....5.....10+	2- . . 5 . . . 10	20 . . 50.....100	200 . . 500+	

END OF SOUNDING

ONTARIO HYDRO - CIVIL RESEARCH - KRCSC
FILE NAME: LARG GIBSON SLOPE STUDY - DECEW HEADPOND
FILE NUMBER: 43198
TEST NO. BORRHOLE - 7
RECORD OF DILATOMETER TEST NO. BORRHOLE - 7
USING DATA REDUCTION PROCEDURES IN MARCHETTI (ASCE, J-GRD, MARCH 80)
EO IN SANDS DETERMINED USING SCHUBERTMANN METHOD (1983)
MODIFIED MAYNE AND KULHAWY FORMULA USED FOR OCR IN SANDS (ASCE, J-GRD, JUNE 82)
LOCATION: ON SECTION 2, 25' FROM SHORE, PUSHED BY HAND, VERTICAL
PERFORMED - DATE: 03 JULY 86
BY: AL MORTON

CALIBRATION INFORMATION:
DELTA A = .14 BARS DELTA B = .70 BARS GAGE 0 = .25 BARS GWT DEPTH = .60 M

1 BAR = 1.019 KG/CM2 = 1.044 TSP = 14.51 PSI ANALYSIS USES H2O UNIT WEIGHT = 1.000 T/M3

Z	THRUST	A	B	BD	ID	ED	UD	GAMMA	SV	PC	OCR	EO	CU	PHI	M	SOIL TYPE
(M)	(KG)	(BAR)	(BAR)	(BAR)	(BAR)	(BAR)	(BAR)	(T/M3)	(BAR)	(BAR)	(BAR)	(BAR)	(BAR)	(DEG)	(BAR)	
4.27		.80	1.60	-1.	-.15		.419	P01 =	.69	P0 =	.69	P1 =	.65			QUESTIONABLE
4.48		.90	1.70	-1.	-.11		.440	P01 =	.79	P0 =	.79	P1 =	.75			QUESTIONABLE

END OF SOUNDING

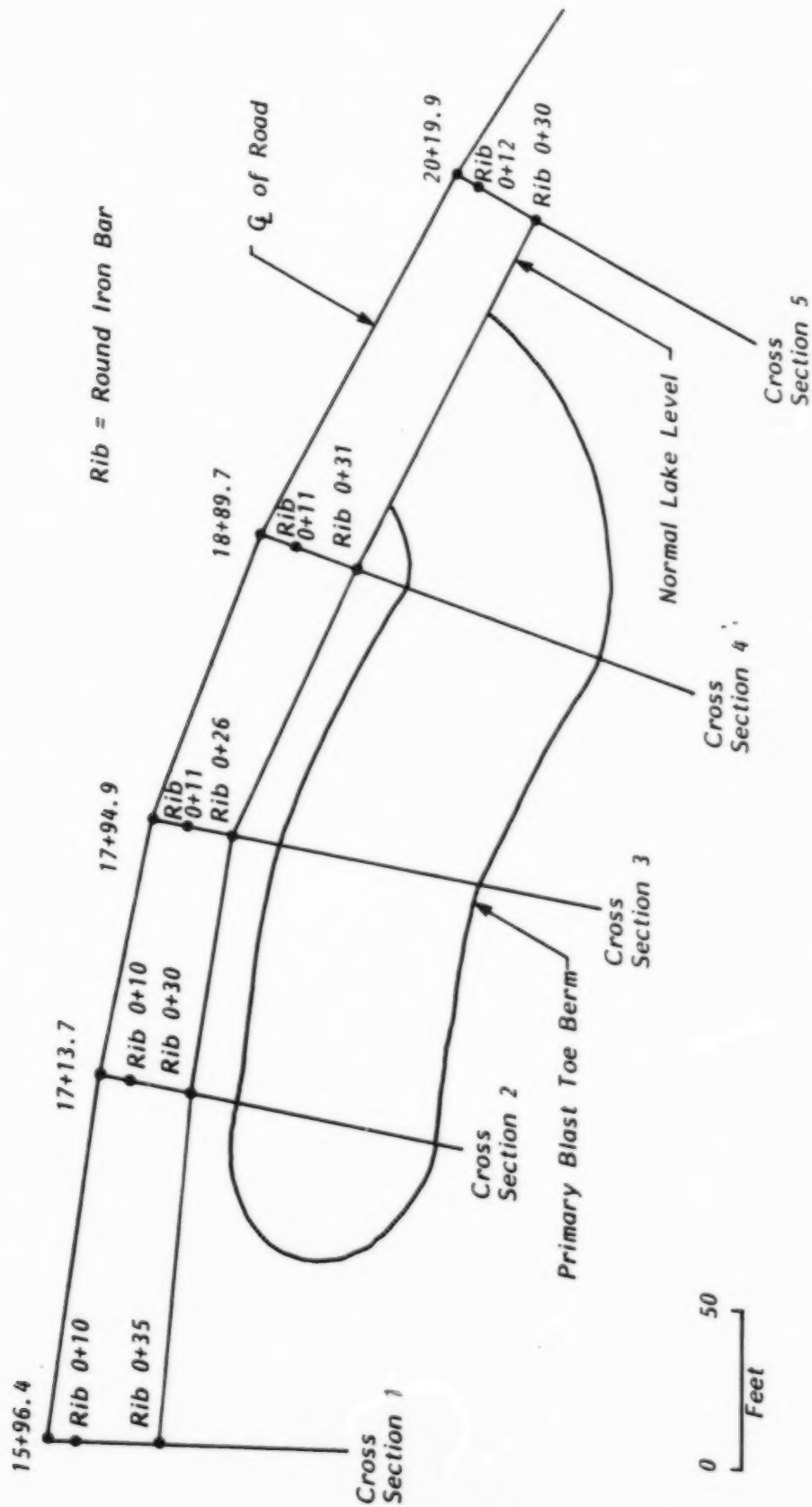


FIGURE 14
PLAN VIEW OF PROPOSED TOE BERM

APPENDIX A

ONTARIO HYDRO - CIVIL RESEARCH - RE252
 FILE NAME: LAKE GIBSON SLOPE STUDY - ECEW HEADPOND
 FILE NUMBER: 43198

TEST NO. BOREHOLE - 1

RECORD OF DILATOMETER TEST NO. BOREHOLE - 1
 USING DATA REDUCTION PROCEDURES IN MARCHETTI (ASCE, J-GED, MARCH 80)
 EO IN SANDS DETERMINED USING SCHMBETMANN METHOD (1983)
 MODIFIED MAYNE AND KULHAWY FORMULA USED FOR OCE IN SANDS (ASCE, J-GED, JUNE 82)

LOCATION: 12' N OF ROAD CL, 10' W OF CABLE DUCT, PUSHED BY HAND, VERTICAL
 PERFORMED - DATE: 03 JULY 86
 BY: AL MORTON

CALIBRATION INFORMATION:

DELTA A = .14 BARS DELTA B = .70 BARS GAGE O = .25 BARS GWT DEPTH = 2.74 M

1 BAR = 1.019 KG/CM2 = 1.044 TSF = 14.51 PSI

ANALYSIS USES H2O UNIT WEIGHT = 1.000 T/M3

Z (M)	THRUST (KG)	A (BAR)	B (BAR)	ED (BAR)	ID	KD	UO (BAR)	GAMMA (T/M3)	SV (BAR)	PC (BAR)	OCE	EO	CU (BAR)	PHI (DEG)	M (BAR)	SOIL TYPE
4.47		5.20	9.80	137.	.83	6.96	.170	1.800	.680	4.76	6.99	1.46	.711		292.3	CLAYEY SILT
4.67		5.50	10.90	133.	.77	7.21	.189	1.800	.696	5.15	7.40	1.49	.761		289.2	CLAYEY SILT
4.88		6.60	11.40	144.	.68	8.54	.210	1.800	.712	6.86	9.63	1.66	.962		337.5	CLAYEY SILT
5.08		6.10	10.80	141.	.73	7.65	.230	1.800	.728	5.90	8.11	1.55	.856		313.3	CLAYEY SILT
5.28		6.00	11.20	159.	.84	7.29	.249	1.800	.744	5.60	7.52	1.50	.824		346.5	CLAYEY SILT
5.49		7.70	16.40	286.	1.19	9.08	.270	1.950	.763	8.08	10.59	1.73			688.6	SILT
5.69		7.40	11.00	101.	.42	8.81	.289	1.800	.779	7.87	10.11	1.70	1.094		238.4	SILTY CLAY
5.99		7.80	10.80	79.	.31	9.15	.309	1.800	.795	8.52	10.73	1.74	1.170		189.6	CLAY
6.10		7.60	10.30	68.	.28	8.71	.330	1.800	.811	8.06	9.93	1.69	1.123		159.9	CLAY
6.30		8.20	14.30	192.	.74	9.01	.349	1.950	.830	8.69	10.47	1.72	1.198		458.3	CLAYEY SILT
6.50		7.80	11.00	86.	.34	8.52	.369	1.800	.845	8.11	9.59	1.66	1.138		200.9	CLAY
6.71		8.20	11.20	79.	.30	8.81	.390	1.800	.862	8.71	10.10	1.70	1.210		186.5	CLAY
6.91		8.60	12.10	97.	.35	9.06	.409	1.800	.878	9.26	10.55	1.73	1.275		232.4	SILTY CLAY
7.11		8.60	11.90	90.	.33	8.89	.429	1.800	.893	9.15	10.24	1.71	1.268		213.2	CLAY
7.32		9.20	12.80	101.	.34	9.32	.449	1.900	.912	10.07	11.04	1.76	1.374		244.2	CLAY
7.52		8.90	11.80	75.	.26	8.86	.469	1.800	.928	9.46	10.19	1.70	1.311		178.3	CLAY
7.72		8.70	11.30	64.	.23	8.50	.489	1.800	.943	9.01	9.55	1.66	1.265		149.6	CLAY
7.92		8.20	10.20	42.	.16	7.85	.508	1.800	.959	8.09	8.43	1.58	1.165		95.2	CLAY
8.13		8.00	10.00	42.	.17	7.49	.529	1.800	.975	7.65	7.84	1.53	1.117		93.1	CLAY
8.33		7.90	9.80	39.	.15	7.25	.549	1.800	.991	7.39	7.46	1.50	1.091		83.8	CLAY
8.53		7.60	9.30	31.	.13	6.83	.568	1.800	1.007	6.84	6.80	1.44	1.029		66.1	CLAY
8.74		8.50	10.40	39.	.14	7.57	.589	1.800	1.023	8.16	7.98	1.54	1.189		85.5	CLAY
8.94		7.20	8.60	20.	.09	6.22	.608	1.700	1.037	6.09	5.88	1.35	.943		41.1	OFF CHART
9.14		6.40	8.50	46.	.24	5.32	.628	1.800	1.053	4.84	4.60	1.21	.786		85.1	CLAY
9.35		6.70	8.40	31.	.15	5.53	.649	1.700	1.067	5.21	4.88	1.25	.837		59.3	CLAY
9.55		6.00	7.40	20.	.11	4.80	.668	1.700	1.081	4.24	3.92	1.13	.711		35.7	CLAY
9.75		6.40	8.30	39.	.20	5.06	.688	1.800	1.097	4.67	4.25	1.17	.770		69.6	CLAY
9.96		6.00	8.20	50.	.28	4.59	.709	1.800	1.113	4.07	3.66	1.09	.692		84.4	CLAY

TEST NO. BOREHOLE - 1

(CONTINUED)

PAGE 1

Z (M)	THRUST (KG)	A (BAR)	B (BAR)	ED (BAR)	ED (BAR)	ED (BAR)	UO (BAR)	GAMMA (T/M3)	SV (BAR)	PC (BAR)	OCB	EO	CU (BAR)	PHI (DEG)	W (BAR)	SOIL TYPE
10.16		7.70	10.70	79.	.34	5.98	.728	1.800	1.129	6.24	5.53	1.32	.977		155.3	CLAY

END OF SOUNDING

DEPTH	X-UNDRAINED SHEAR STRENGTH (CU) - BARS				X-PRECONSOLIDATION PRESSURE (PC) - BARS				X-MODULUS FOR 1-D CONSOLIDATION (M) - BARS (LOGARITHMIC SCALE)						
	0.....1.....2+				0.....5.....10+				2- . . 5 . . . 10	20 . . 50....100	200 . . 500+				
4.00 M	+	+	+	+	+	+	+	+	+	+	+	+	+	+	15.1 FT
4.20 M															13.8 FT
4.47 M		X			X						X				14.7 FT
4.67 M		X			X						X				15.3 FT
4.88 M		X			X						X				16.0 FT
5.08 M	+	+	X	+	+	+	X	+	+	+	+	+	X	+	16.7 FT
5.28 M		X			X						X				17.3 FT
5.49 M							X					X			18.0 FT
5.69 M		X			X						X				18.7 FT
5.89 M		X			X						X				19.3 FT
6.10 M	+	+	X	+	+	+	X	+	+	+	+	+	X	+	20.0 FT
6.20 M															20.3 FT
6.30 M		X			X							X			20.7 FT
6.50 M		X			X						X				21.3 FT
6.71 M		X			X						X				22.0 FT
6.91 M	+	+	X	+	+	+	X	+	+	+	+	+	X	+	22.7 FT
7.11 M		X			X						X				23.3 FT
7.32 M		X			X						X				24.0 FT
7.52 M		X			X						X				24.7 FT
7.72 M		X			X						X				25.3 FT
7.92 M	+	+	X	+	+	+	X	+	+	+	+	+	X	+	26.0 FT
8.13 M		X			X						X				26.7 FT
8.33 M		X			X						X				27.3 FT
8.53 M		X			X						X				28.0 FT
8.74 M		X			X						X				28.7 FT
8.94 M	+	+	X	+	+	+	X	+	+	+	+	+	X	+	29.3 FT
9.14 M		X			X						X				30.0 FT
9.35 M		X			X						X				30.7 FT
9.55 M		X			X					X					31.3 FT
9.75 M		X			X						X				32.0 FT
9.96 M	+	+	X	+	+	+	X	+	+	+	+	+	X	+	32.7 FT
10.16 M		X			X						X				33.3 FT
10.40 M															34.1 FT
10.60 M															34.8 FT
10.80 M	+	+	+	+	+	+	+	+	+	+	+	+	+	+	35.4 FT
	25-..30...35...40...45+				0.....5.....10+				2- . . 5 . . . 10	20 . . 50....100	200 . . 500+				
	O-FRICTION ANGLE (PHI) - DEG				X-VERTICAL EFFECTIVE STRESS (SV) - BARS										

END OF SOUNDING

ONTARIO HYDRO - CIVIL RESEARCH - RE252
 FILE NAME: LAKE GIBSON SLOPE STUDY - DECEW HEADPOND
 FILE NUMBER: 43198

TEST NO. BOREHOLE - 3

RECORD OF DILATOMETER TEST NO. BOREHOLE - 3
 USING DATA REDUCTION PROCEDURES IN MARCHETTI (ASCE, J-GED, MARCH 80)
 K0 IN SANDS DETERMINED USING SCHMERTMANN METHOD (1983)
 MODIFIED MAYNE AND KULHAWY FORMULA USED FOR OCR IN SANDS (ASCE, J-GED, JUNE 82)

LOCATION: ON SECTION 4, 25' FROM SHORE, PUSHED BY HAND, VERTICAL
 PERFORMED - DATE: 03 JULY 86
 BY: AL MORTON

CALIBRATION INFORMATION:

DELTA A = .14 BARS DELTA B = .70 BARS GAGE 0 = .25 BARS GWT DEPTH = .00 M

1 BAR = 1.019 KG/CM2 = 1.044 TSF = 14.51 PSI

ANALYSIS USES H2O UNIT WEIGHT = 1.000 T/M3

Z (M)	THRUST (KG)	A (BAR)	B (BAR)	BD (BAR)	ID	ED	UO (BAR)	GAMMA (T/M3)	SV (BAR)	PC (BAR)	OCR	K0	CU (BAR)	PHI (DEG)	N (BAR)	SOIL TYPE
1.93	.60	1.40	-1.	-.13			.189	P01 =	.49	P0 =	.49	P1 =	.45			QUESTIONABLE
1.94	.60	1.50	2.	.21			.190	P01 =	.49	P0 =	.49	P1 =	.55			QUESTIONABLE
1.95	.90	1.80	2.	.11			.191	P01 =	.49	P0 =	.79	P1 =	.85			QUESTIONABLE

END OF SOUNDING

DEPTH	X-UNDRAINED SHEAR STRENGTH (CU) - BARS	X-PRECONSOLIDATION PRESSURE (PC) - BARS	X-MODULUS FOR 1-D CONSOLIDATION (M - BARS (LOGARITHMIC SCALE)				
	0.....1.....2+	0.....5.....10+	2- . . 5 10	20 . . 50....100	200 . .500+		
1.00 M	+-----+-----+	+-----+-----+	+-----+-----+	+-----+-----+	+-----+-----+		3.3 FT
1.20 M							3.9 FT
1.40 M							4.6 FT
1.60 M							5.2 FT
1.80 M	+-----+-----+	+-----+-----+	+-----+-----+	+-----+-----+	+-----+-----+		5.9 FT
	25-..30...35...40..45+	0.....5.....10+	2- . . 5 10	20 . . 50....100	200 . .500+		
	0-FRICTION ANGLE (PHI) - DEG	*-VERTICAL EFFECTIVE STRESS (SV) - BARS					

END OF SOUNDING

ONTARIO HYDRO - CIVIL RESEARCH - KR252
FILE NAME: LAKE GIBSON SLOPE STUDY - DECEW HEADPOND
FILE NUMBER: 43198

TEST NO. BOREHOLE - 4

RECORD OF DILATOMETER TEST NO. BOREHOLE - 4
USING DATA REDUCTION PROCEDURES IN MARCHETTI (ASCE, J-GED, MARCH 80)
K0 IN SANDS DETERMINED USING SCHMERTMANN METHOD (1983)
MODIFIED MAYNE AND KULHAWY FORMULA USED FOR OCR IN SANDS (ASCE, J-GED, JUNE 82)

LOCATION: ON SECTION 4, 75' FROM SHORE, PUSHED BY HAND, VERTICAL
PERFORMED - DATE: 03 JULY 86
BY: AL MORTON

CALIBRATION INFORMATION:

DELTA A = .14 BARS DELTA B = .70 BARS GAGE 0 = .25 BARS GWT DEPTH = .00 M

1 BAR = 1.019 KG/CM2 = 1.044 TSP = 14.51 PSI

ANALYSIS USES H2O UNIT WEIGHT = 1.000 T/M3

Z (M)	THRUST (KG)	A (BAR)	B (BAR)	ED (BAR)	ID	ED	UO (BAR)	GAMMA (T/M3)	SV (BAR)	PC (BAR)	OCR	K0	CU (BAR)	PHI (DEG)	M (BAR)	SOIL TYPE
4.62		.90	1.70	-1.	-.12		.453	P01 =	.79	P0 =	.79	P1 =	.75			QUESTIONABLE
.00		.00	.00	-29.	7.64		.000	P01 =	-.07	P0 =	-.11	P1 =	-.95			QUESTIONABLE

END OF SOUNDING

X-UNDRAINED SHEAR
 STRENGTH (CU) - BARS
 DEPTH 0.....1.....2+
 25-..30...35...40..45+
 O-FRICTION ANGLE
 (PHI) - DEG

X-PRECONSOLIDATION
 PRESSURE (PC) - BARS
 0.....5.....10+
 0.....5.....10+
 *-VERTICAL EFFECTIVE
 STRESS (SV) - BARS

X-MODULUS FOR 1-D CONSOLIDATION M_v - BARS
 (LOGARITHMIC SCALE)
 2- . . 5 10 20 . . 50....100 200 . .500+
 2- . . 5 10 20 . . 50....100 200 . .500+

BND OF SOUNDING

ONTARIO HYDRO - CIVIL RESEARCH - KB25C
 FILE NAME: LAKE GIBSON SLOPE STUDY - DECEW HEADPOND
 FILE NUMBER: 43198

TEST NO. BOREHOLE - 5

RECORD OF DILATOMETER TEST NO. BOREHOLE - 5
 USING DATA REDUCTION PROCEDURES IN MARCHETTI (ASCE, J-GED, MARCH 80)
 EO IN SANDS DETERMINED USING SCHMERTMANN METHOD (1983)
 MODIFIED MAYNE AND KULHAWY FORMULA USED FOR OCR IN SANDS (ASCE, J-GED, JUNE 82)

LOCATION: ON SECTION 3, 25' FROM SHORE, PUSHED BY HAND, VERTICAL
 PERFORMED - DATE: 03 JULY 86
 BY: AL MORTON

CALIBRATION INFORMATION:

DELTA A = .14 BARS DELTA B = .70 BARS GAGE 0 = .25 BARS GWT DEPTH = .00 M

1 BAR = 1.019 KG/CM2 = 1.044 TSF = 14.51 PSI

ANALYSIS USES H2O UNIT WEIGHT = 1.000 T/M3

Z (M)	THRUST (KG)	A (BAR)	B (BAR)	ED (BAR)	ID (BAR)	ED (BAR)	UO (BAR)	GAMMA (T/M3)	SV (BAR)	PC (BAR)	OCR (BAR)	EO (BAR)	CU (BAR)	PHI (DEG)	M (BAR)	SOIL TYPE
5.13		1.00	1.90	2.	.16	.77	.503	1.500	.500	.11	.22	.13	.033		1.9	MUD
.00		.00	.00	-29.	7.54		.000	PO1 =	-.07	PO =	-.11	PI =	-.95			QUESTIONABLE

END OF SOUNDING

	X-UNDRAINED SHEAR STRENGTH (CU) - BARS	X-PRECONSOLIDATION PRESSURE (PC) - BARS	X-MODULUS FOR 1-D CONSOLIDATION (M) - BARS (LOGARITHMIC SCALE)			
DEPTH	0.....1.....2+	0.....5.....10+	2- . . 5 . . . 10	20 . . 50 . . . 100	200 . . 500+	
	25...30...35...40..45+	0.....5.....10+	2- . . 5 . . . 10	20 . . 50 . . . 100	200 . . 500+	
	O-FRICTION ANGLE (PHI) - DEG	*-VERTICAL EFFECTIVE STRESS (SV) - BARS				

END OF SOUNDING

ONTARIO HYDRO - CIVIL RESEARCH - ER250
FILE NAME: LAKE GIBSON SLOPE STUDY - DECEW HEADPOND
FILE NUMBER: 43198

TEST NO. BOREHOLE - 6

RECORD OF DILATOMETER TEST NO. BOREHOLE - 6
USING DATA REDUCTION PROCEDURES IN MARCHETTI (ASCE, J-GED, MARCH 80)
K0 IN SANDS DETERMINED USING SCHNEIBTMANN METHOD (1983)
MODIFIED MAYNE AND KULHAWY FORMULA USED FOR OCR IN SANDS (ASCE, J-GED, JUNE 82)

LOCATION: ON SECTION 3, 75' FROM SHORE, PUSHED BY HAND, VERTICAL
PERFORMED - DATE: 03 JULY 86
BY: AL MORTON

CALIBRATION INFORMATION:

DELTA A = .14 BARS DELTA B = .70 BARS GAGE C = .25 BARS GWT DEPTH = .00 M

1 BAR = 1.019 KG/CM2 = 1.044 TSF = 14.51 PSI

ANALYSIS USES H2O UNIT WEIGHT = 1.000 T/M3

Z (M)	THRUST (KG)	A (BAR)	B (BAR)	ED (BAR)	ID (BAR)	ED (BAR)	UO (BAR)	GAMMA (T/M3)	SV (BAR)	PC (BAR)	OCR	K0	CU (BAR)	PHI (DEG)	M (BAR)	SOIL TYPE
5.23		1.00	1.90	2.	.17	.73	.513	1.500	.510	.11	.21	.11	.032		1.9	MUD
5.44		1.00	1.90	2.	.18	.68	.534	1.500	.520	.10	.19	.09	.030		1.9	MUD

END OF SOUNDING

DEPTH	X-UNDRAINED SHEAR STRENGTH (CU) - BARS	X-PRECONSOLIDATION PRESSURE (PC) - BARS	X-MODULUS FOR 1-D CONSOLIDATION M. - BARS (LOGARITHMIC SCALE)			
	0.....1.....2+	0.....5.....10+	2- . . 5 . . . 10	20 . . 50 . . . 100	200 . . 500+	
4.00 M	+-----+-----+	+-----+-----+	+-----+-----+	+-----+-----+	+-----+-----+	13.1 FT
4.20 M						13.8 FT
4.40 M						14.4 FT
4.60 M						15.1 FT
4.80 M	+-----+-----+	+-----+-----+	+-----+-----+	+-----+-----+	+-----+-----+	15.7 FT
	25...30...35...40...45+	0.....5.....10+	2- . . 5 . . . 10	20 . . 50 . . . 100	200 . . 500+	
	0-FRICTION ANGLE (PHI) - DEG	*-VERTICAL EFFECTIVE STRESS (SV) - BARS				

END OF SOUNDING

ONTARIO HYDRO - CIVIL RESEARCH - 82552
 FILE NAME: LAKE GIBSON SLOPE STUDY - DRCEW HEADPOND
 FILE NUMBER: 43198

TEST NO. BOREHOLE - 8

RECORD OF DILATOMETER TEST NO. BOREHOLE - 8
 USING DATA REDUCTION PROCEDURES IN MARCHETTI (ASCE, J-GED, MARCH 80)
 EO IN SANDS DETERMINED USING SCHNEERTMANN METHOD (1983)
 MODIFIED MAYNE AND EULHAWY FORMULA USED FOR OCR IN SANDS (ASCE, J-GED, JUNE 82)

LOCATION: ON SECTION 2, 75' FROM SHORE, PUSHED BY HAND, VERTICAL
 PERFORMED - DATE: 03 JULY 86
 BY: AL MORTON

CALIBRATION INFORMATION:

DELTA A = .14 BARS DELTA B = .70 BARS GAGE 0 = .25 BARS GWT DEPTH = .90 M

1 BAR = 1.019 KG/CM2 = 1.044 TSF = 14.51 PSI

ANALYSIS USES H2O UNIT WEIGHT = 1.000 T/M3

Z (M)	THRUST (KG)	A (BAR)	B (BAR)	SD (BAR)	ID (BAR)	EO (BAR)	UO (BAR)	GAMMA (T/M3)	SV (BAR)	PC (BAR)	OCR (BAR)	EO (BAR)	CU (BAR)	PHI (DEG)	M (BAR)	SOIL TYPE
5.18	.90	1.70	-1.	-.14		.508	P01 = .79	P0 = .79	P1 = .75							QUESTIONABLE
5.39	1.00	1.70	-5.	-.39		.529	P01 = .90	P0 = .89	P1 = .75							QUESTIONABLE

END OF SOUNDING

DEPTH	X-UNDRAINED SHEAR STRENGTH (CU) - BARS	X-PRECONSOLIDATION PRESSURE (PC) - BARS	X-MODULUS FOR 1-D CONSOLIDATION (M) - BARS (LOGARITHMIC SCALE)				
	0.....1.....2+	0.....5.....10+	2- . . 5 10	20 . . 50....100	200 . .500+		
5.00 M	+	+	+	+	+	+	16.4 FT
5.20 M	:	:	:	:	:	:	17.1 FT
5.40 M	:	:	:	:	:	:	17.7 FT
5.60 M	:	:	:	:	:	:	18.4 FT
5.80 M	+	+	+	+	+	+	19.0 FT
	25-..30...35...40..45+	0.....5.....10+	2- . . 5 10	20 . . 50....100	200 . .500+		
	0-FRICTION ANGLE (PHI) - DEG	*-VERTICAL EFFECTIVE STRESS (SV) - BARS					

BND OF SOUNDING